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4. Design of substructure

4.1 Foundation of the East Bridge

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Introduction

This chapter gives an overview of the East Bridge project, the geotechnical investigation strategy employed, and the particular challenges experienced by the geotechnical engineers in the course of the project. One of the lessons learned from the East Bridge are that collaboration between geotechnical and civil engineers, as well as geologists, is a prerequisite for cost-effective and safe solutions, as well as for advancements in the state of the art.

This chapter deals with the founding of the East Bridge, where the interplay between various disciplines, test types, and numerical approaches played a major part in achieving a safe foundation design. Experiences from the construction of the other parts of the Link have also been implemented.

Investigation strategy

Previous knowledge and its implementation

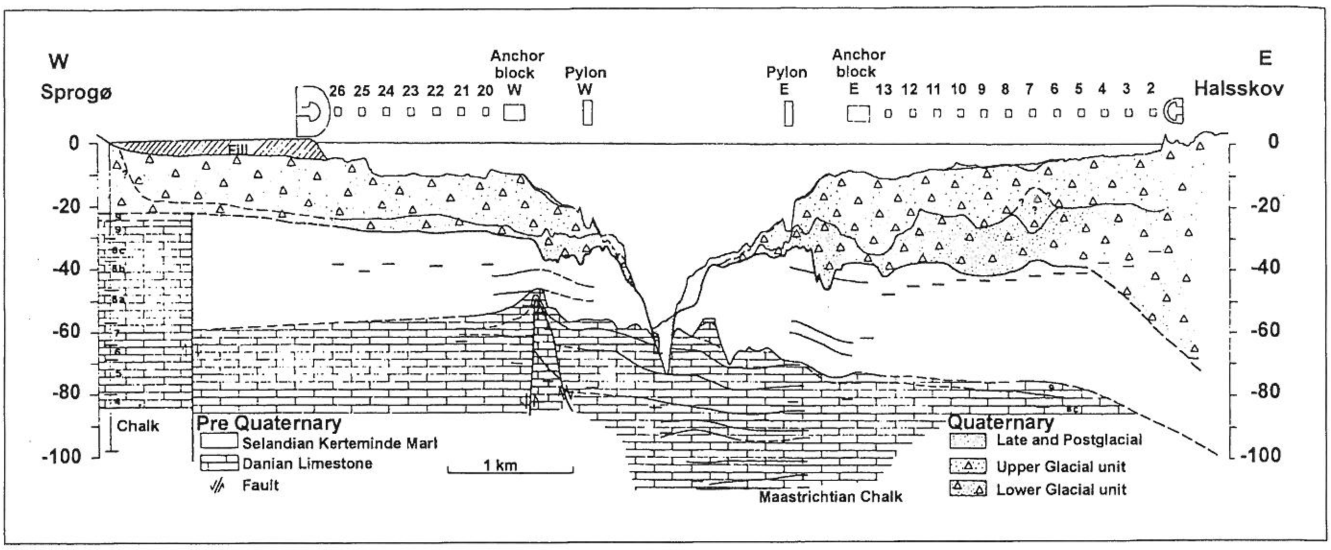
Preliminary soil investigations for a fixed link across the Great Belt were carried out in 1962–63. These were followed by a more detailed campaign in 1977–78 during the design activities of Statsbroen Store Bælt. In 1983 two deep borings were carried out into the marl in connection with a feasibility study for a tunnel solution. The most recent campaign for the link to be built started in 1987 concurrently with investigations at the proposed location, during which the alignment corridor for the bridge between the islands of Sprogø and Zealand was finally established.

The investigations comprised seismic profiling, borings, and laboratory testing, as well as geological studies for the top 100m of the deposits. In 1987 the results, in the form of soil boundaries and geological layer characteristics, were implemented in a newly-established 3D computer database, the Geomodel, established by Storebælt for use and access by the client (management), the consultants (updating and design) and the contractors (construction).

The ground conditions for the East Bridge were thus fairly well known in advance, allowing the alignment as well as the foundation type to be determined. The foundation solutions could be proven by detailed investigations performed simultaneously with the detailed design.

Overview of geology

A comprehensive geological model, based on the investigations up to 1979 and relevant for the East Bridge, was presented in 1982 by Professor Gunnar Larsen from Aarhus University. Some 10 000 years ago the Great Belt area was dry land and the deep Eastern Channel was a north-flowing river, eroded into the



▲ *Fig. 4.1-1 Schematic geological section in East Bridge alignment*

indigenous deposits and with early melt water deposits. Depressions were filled with peat and mud. The Eastern Channel of the Great Belt was flooded during the Flandrian transgression, starting some 8000 years ago, and minor marine sediments were deposited.

The predominant feature for the foundations is two glacial till units originating from two main glaciation periods, the Weichsel and the Saale. Only in the deep channel are these tills missing. The glacial sequence is some 20m thick, increasing to over 70m nearer Zealand. During the Weichsel glaciation the upper till unit was deposited close to the ice border line, presumably some 13 000 years ago. The lower till unit originates from the Saale glaciation and is believed to be between 120 000 and 140 000 years old. The pre-quaternary deposits consist of Selandian marl, known as the Kerteminde-mergel, resting on Danian limestone and at greater depth, the Maastrichtian chalk. The thickness of the Kertemindemergel is some 40metres.

The prevailing geological conditions for the Great Belt project, in the light of all investigations up to 1995, are summarised in the schematic geological section in the East Bridge alignment shown in Figure 4.1-1.

East Bridge investigations

Prior to the detailed investigations for the East Bridge, important experience was gained from the West Bridge detailed investigations and early construction activities, namely:

- The Geomodel was upgraded in 1989 to an intelligent database with storage facilities for geotechnical parameters
- The cone penetration test (CPT) technique had proven very useful for detailed mapping of ground conditions at each pier location as a supplement to regular geotechnical borings
- The importance of field tests such as large plate load testing had been realised and initiated at the island of Sprogø
- The tills had proven to exhibit much more varying strength properties than ever anticipated
- Direct foundation on compacted gravel pads was feasible
- Onshore prefabrication of elements could minimise marine construction.

For the detailed investigations in 1990–92, the following programme was established.

General

The aim of the investigation was to establish a proper 3D geological and geotechnical knowledge of the subsoils down to the depth affected by each structural element. A total of 100 geotechnical borings were to be taken to some 30–40m below the seabed, with a typical spacing of 20–40m at structural element positions. These borings were to include sampling, vane testing, and SPT. In selected boreholes, mainly at the pylon and anchor blocks, pressure meter tests were to be carried out. Some 400 CPTs were carried out to 20m below the seabed, with a typical spacing of 10–20m at structural element positions.

Laboratory testing was to include classification testing as well as strength and deformation testing. Small-scale testing was to be supplemented by large-scale triaxial and shear testing of soils and crushed rock gravel.

The sliding issues for anchor blocks and for piers during ship impacts would be clarified by performing large-scale sliding plate load tests onshore at Zealand on intact as well as remoulded clay tills.

CPT tests

Particular attention was paid to the analysis and numerical filtering of CPT tests in the clay till, drawing on the extensive experience gained from the West Bridge testing. The clay till is a mixture of clay, silt, sand, and gravel, whose general behaviour is governed by the clay. The effects from sand and gravel traces on the CPT tests were considered an irrelevancy to be filtered out, and for each CPT test the spikes pertaining to sand and gravel were removed with a moving average filter. Then a plot was produced displaying all the resulting CPT q_c profiles. If they demonstrated the same general trend, a set of three plots was produced to show (see Figure 4.1-2):

- Collated profiles of average q_c and average of the minima q_c profiles
- A plot of the in situ vane shear tests
- A plot of the water contents, w, and activity, A, from the laboratory tests

The average of minima plot was intended to represent a conservatively estimated mean value, as required by the Danish Code of Practice, DS 415 (1984). If a varying picture of soil appeared, more than one profile was elaborated to provide fully representative soil profiles for the pier. Any yet more careful estimate was not considered necessary, since any failure would have to pass through the weaker as well as the stronger parts of the soil.

From these tests and the geological descriptions of samples from the geotechnical borings, a characteristic design soil profile could be produced. This profile was used as the basis for the foundation design.

Laboratory test scheme, intact clay till

A general pattern of behaviour in terms of relations between strength, preconsolidation stress and in situ vertical stress appeared when SHANSEP was applied to the laboratory test data.

Advanced, high quality laboratory tests were performed on a select array of intact samples covering the widest possible spectrum of in situ strengths and stresses, rather than on specimens from every pier. The undrained shear strength of the clay till was determined by triaxial compression ($c_{u,C}$) and extension ($c_{u,E}$) tests as well as by volume constant direct shear tests ($c_{u,DS}$).

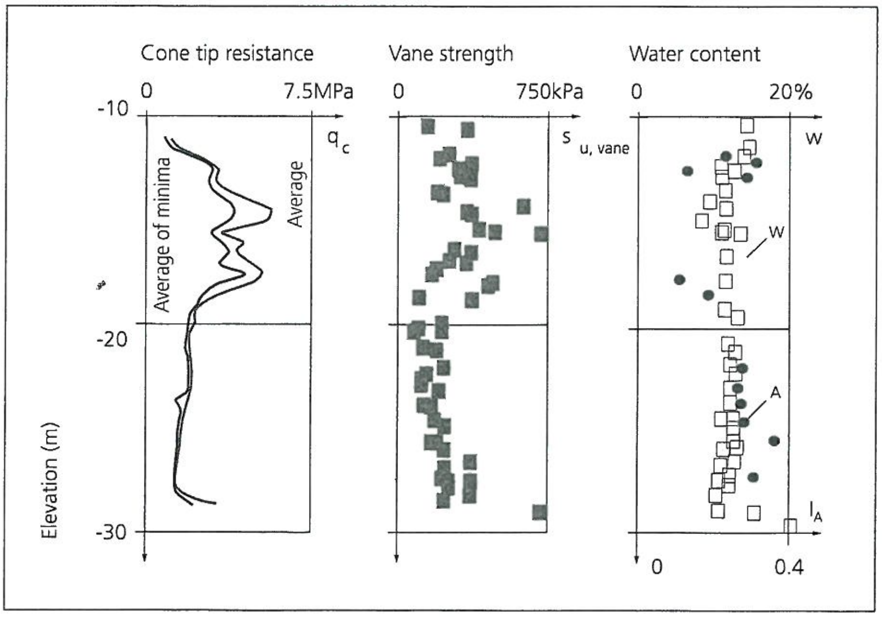
The triaxial tests were carried out on 70mm diameter specimens with a height/diameter ratio of 1:1 and smooth pressure heads, to ensure homogeneous strain conditions. To produce reliable results, the in situ stress history was reproduced by anisotropic consolidation to the preconsolidation stress, followed by unloading to the in situ stress prior to the load test to failure.

The direct shear tests were carried out in an automated, very rigid purpose-built shear box (DAS). The specimens were cylindrical, 70mm diameter, with a height of 30mm. The tests were carried out as volume constant tests in order to simulate undrained conditions.

The suite of laboratory tests was compared to the field values of undrained shear strength. They consisted of $s_{u,vane}$, from field vane tests in the boring and $s_{u,CPT} = q_c/N_k$ inferred from CPT tests carried out close to the relevant bore hole (s_u is here used for the in situ determined undrained shear strength as opposed to c_u in the laboratory). The relationship between $s_{u,vane}$ and q_c was based on the correlation of all vane and CPT tests carried out in the clay till in the Eastern Channel area. The following strength model was found to apply for the intact clay till:

$$\begin{aligned}
c_{u,C} &= 0.42\,\sigma'_v\,(\sigma'_{pc}/\sigma'_v)^{0.85} \\
c_{u,DS} &= 0.86\,c_{u,C} \\
c_{u,E} &= 0.71\,c_{u,C} \\
s_{u.vane} &= 0.88\,c_{u,C} \\
s_{u.CPT} &= q_c / 10
\end{aligned} \tag{1}$$

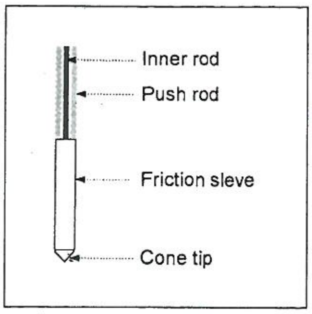
► *Fig. 4.1-2 Example of background plots for design soil profile from an approach pier*



Probabilistic soil strength model

Compared with traditional boring and sampling, the CPT method generates a large amount of data efficiently and economically. However, due to limited experience with the use of CPTs in clay till, a well-documented relationship between the cone resistance record and the relevant geotechnical parameters was not available.

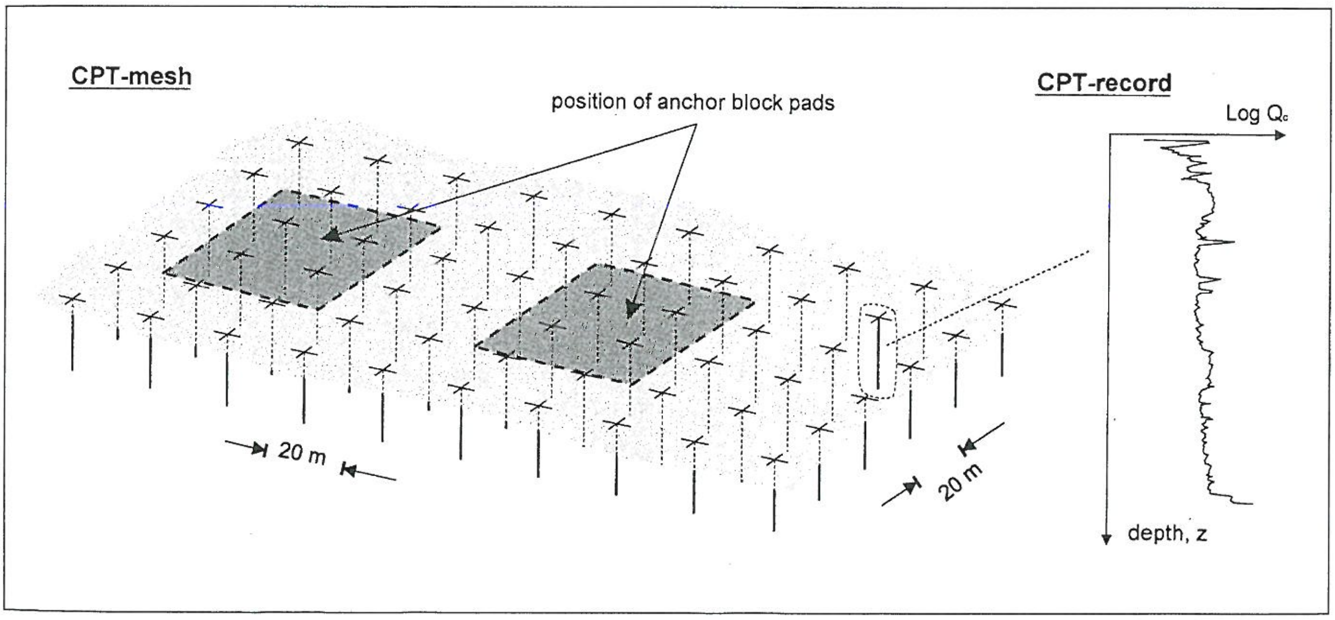
Professor Ove Ditlevsen of the Technical University of Denmark proposed and developed an ambitious solution which handles the uncertainties of this transformation explicitely, and utilise the vast amount of information in the CPT records. When combined with a detailed soil failure model, the reliability of the foundations can be assessed with full use of all the available information.



▲ *Fig. 4.1-3 CPT-pentrometer*

CPT field model

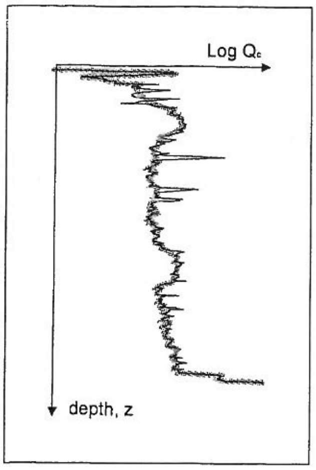
A total of 60 vertical CPT records cover the soil volume below each anchor block foundation. The CPTs are arranged in a uniform 20m mesh, covering a total area of 180m by 100m (see Figure 4.1-4). Each CPT record contains samples of the cone resistance at 20mm intervals, and typically covers 12–14m of clay till. Hence, the CPT test programme provides a total of 3500–4000 samples for each anchor block.



▲ *Fig. 4.1-4 CPT programme for the anchor blocks*

These samples are represented as a realisation of a homogeneous, three-dimensional, stochastic field, denoted the CPT field. The field is assumed to be log normal (ie the exponential of a normal field), which ensures that only positive values are represented. Different correlation structures are assumed in the vertical and horisontal direction, due to the horizontal stratification of the soil volume.

CPT records provide almost ideal point information, which is an advantage in relation to the theoretical model but entails practical problems due to the presence of stones in the inhomogeneous clay till. The stones are sparse so the geotechnical properties of the till are governed by the weak clay matrix. However, when the penetrometer hits a stone, the record will develop an abrupt, local peak, and thus falsely indicate a higher soil strength. These errors in the records due to stones are eliminated from the CPT records by using the assumed vertical correlation structure.



▲ *Fig. 4.1-5 Smoothing of CPT record*

This 'smoothing' of the records is performed in a statistical sense using the maximum likelihood principle. A typical example is shown in Figure 4.1-5.

The CPT field model represents the smoothed records exactly and specifies distributions for the field parameters: these are the mean, variance, and correlation structure parameters, and in essence represent a stochastic interpolation between the CPT records.

Conversion to the field of un-drained shear strength

Earlier investigations suggested the CPT measurement in clay till to be proportional to traditional vane test measurements, although some random deviation or 'noise' was observed. Combined with the general confidence in a linear relationship between vane test results in clay till and the undrained shear strength, the CPT field was assumed to be proportional to the undrained shear strength. In addition, observed deviations from this hypothesis are recognized and explicitely modelled.

The formulation is calibrated using vane test results performed in traditional borings in the region covered by the CPT field. The distribution of the CPT field at the positions of the vane tests provides the information to develop an a posteriori distribution of the proportionality factor and the measuring uncertainty.

Consolidation

A final refinement of the soil strength model was to include the strength increase due to the consolidating effect of the anchor block loading. Based on laboratory triaxial tests, a model for the strength increase was suggested.

Particular challenges

Foundation principles

The main geotechnical challenge was the mere size of the project, which was beyond normal experience and codes of practice, and led to very thorough investigations as well as very careful independent assessments of the safety philosophy.

The quality of the design was optimised by applying completely independent design models for all key issues.

The main issues to be considered were the sliding stability of the anchor blocks and ship impact on piers close to the heavily trafficked navigation channel.

Throughout the long pre-history of this bridge it was always expected that the principle should be that of a direct shallow foundation. Based on experience with the West Bridge it was decided that all footings should rest on compacted gravel pads of crushed rock. Considering their extraordinarily large dimensions, all caissons had to be manufactured with a skirt system designed for penetration into the gravel pad, and thus establishing an ambient space for grouting.

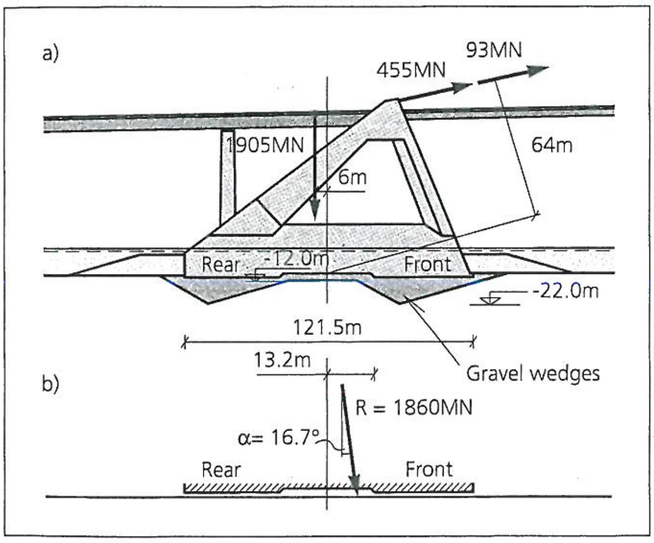
Anchor blocks

Each of the two anchor blocks has a rectangular base 121.5m long and 54.5m wide (see Figure 4.1-6). This base is divided into three parts, a front pad of 41.7m, a middle part of 39.1m, and a rear pad of 40.7m. Only the front and the rear pads are in contact with the supporting soils. Both anchor blocks are founded on very stiff to hard preconsolidated clay till. The undrained shear strengths range from 150 to 300kPa.

As a result of excavation, the top part of the clay till was expected to be disturbed and to have a reduced sliding resistance. This problem was compensated for by introducing a wedge-shaped fill of compacted crushed stone below each of the two pads.

Anchor block loads

The loading situation shown in Figure 4.1-6 under a) leads to a resultant force (under b). Assuming a uniform vertical load distribution for each of the two pads, the two vertical reaction forces are statically determined.



◄ *Fig. 4.1-6*
(a) Elevation of anchor block;
(b) Load resultant acting at foundation level

The horizontal shear load can be assumed as distributed in such a way that the two foundation pads have the same safety against bearing capacity failure. This assumption is not necessary with a finite element analysis where the total structure is analysed and where the horizontal shear load is distributed automatically. This, of course, implies that the concrete superstructure has the rigidity and the strength needed to distribute the shear loading.

Bearing capacity analysis

The bearing capacity analysis was performed both as a traditional deterministic and as an advanced probabilistic in the ultimate limit state analysis. The partial factors used for the deterministic analysis are summarised in Table 4.1-1.

Table 4.1-1 Partial factors applied in deterministic analysis		
Quantity	Partial factor	Value
Permanent action	γ_G	1.0
Variable action	γ_Q	1.3
Tangent of internal angle of friction	γ_φ	1.3
Undrained shear strength	γ_c	2.0

Three different principal types of failure mode are possible for each foundation pad, depending on load inclination, as shown in Figure 4.1-7.

The critical mode for a given case will depend upon geometry, soil strength, and the inclination of the resultant force.

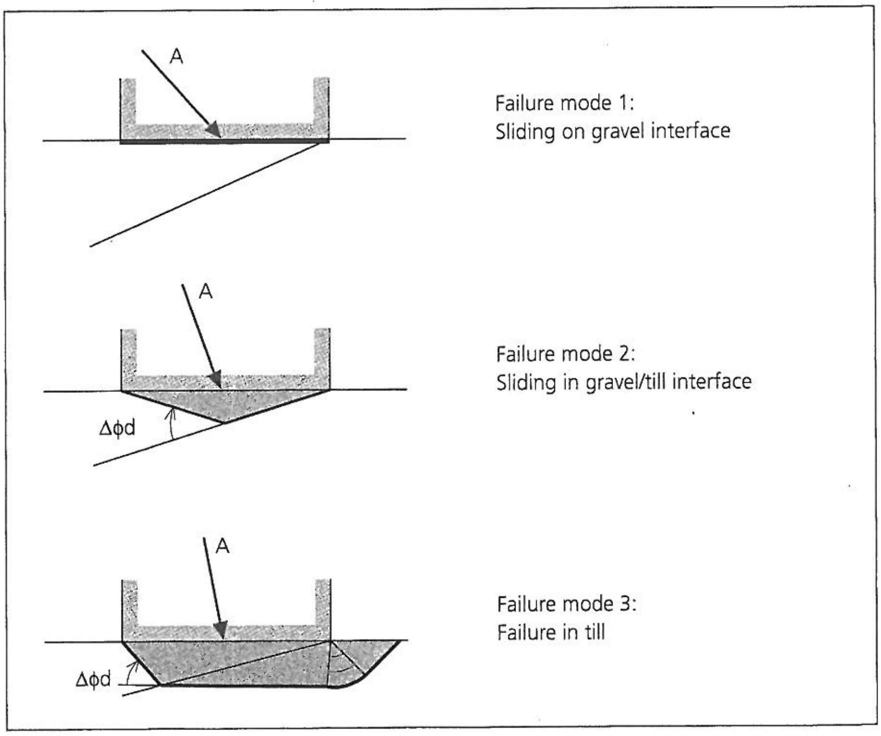
A correct solution for the bearing capacity has to be both statically and kinematically admissible. It is, however, difficult to find solutions which fulfil both conditions, so it was decided to use three different and independent calculation methods to determine the bearing capacity of the anchor blocks. The selected methods, all in 2D, included:

- Upper bound theory
- Limit equilibrium analysis (BEAST)
- Finite element analysis (ABAQUS).

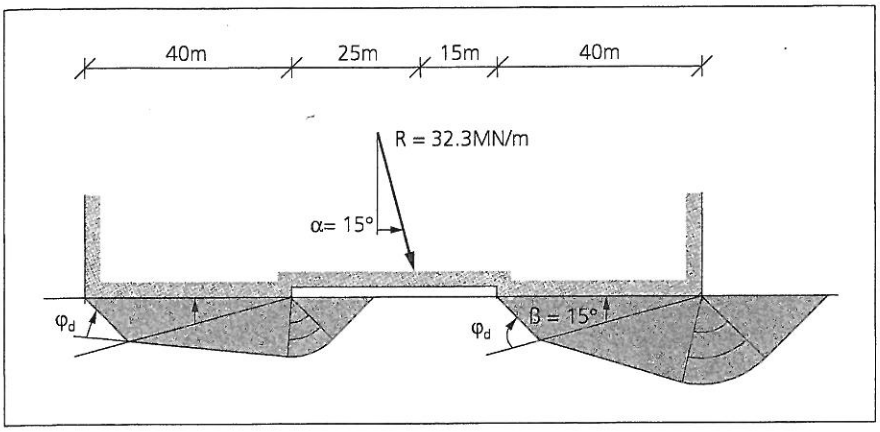
Prior to calculation of the bearing capacity of the anchor blocks, the three calculation methods were tested through five benchmark cases. From the results it was concluded that all three methods could be considered as relevant and usable tools for the design procedure.

One of the tests performed, the anchor block case, had a geometry and a soil strength nearly identical to those of the real anchor block. The calculated resulting bearing capacities were found to range from 32.3MN/m to 33.8MN/m (corresponding to the 2D simplification of strip footings). The rupture figures for the upper bound and ABAQUS analyses are shown in Figures 4.1-8 and 4.1-9.

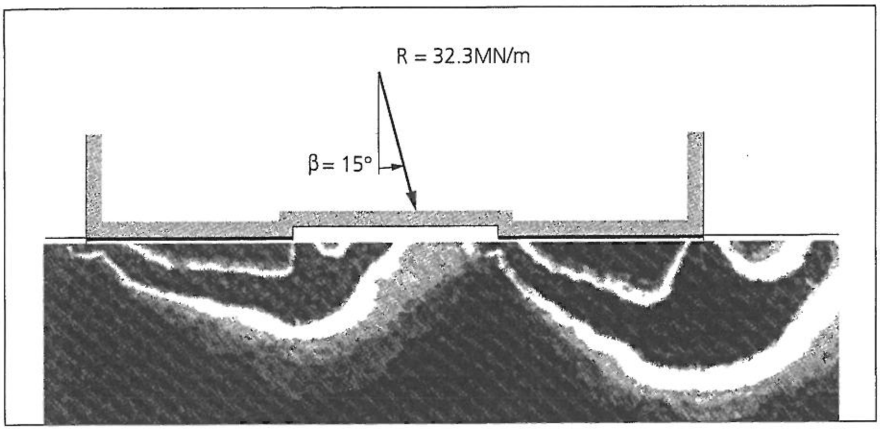
► *Fig. 4.1-7 Foundation pad failure modes*



Supplementary 3D analyses were also performed to verify these 2D analyses.



◄ *Fig. 4.1-8 Rupture figure, upper bound analysis*



◄ *Fig. 4.1-9 Rupture figure, ABAQUS analysis*

Soil strength

Substantial efforts were launched to determine the strength of the soils involved. These efforts have led to a degree of confidence corresponding to the complexity of this structure in agreement with DS 415, High Foundation Class.

Crushed stone in gravel wedges

The crushed stone is a Hyperite quarried from Kragero, Norway. The grains are approximately disc-shaped with a high grain density of $\rho_s = 3.12 Mg/m^3$, a high compression strength of σ_c = 150MPa – 200MPa, and a mean value of 170MPa. The grain size curves for the compacted material showed a mean grain size of $d_{50} \approx$ 15mm and a uniformity coefficient of $C_U \approx 10$. Triaxial tests, carried out in a large, purpose-built, triaxial set-up with cylindrical specimens 500mm × 500mm, indicated very high values of the triaxial secant angle of friction, $\Phi > 50°$. In all the tests the material dilated at failure, but the rate of dilatancy was lower than expected.

The upper bound bearing capacity calculation implicitly assumes that the angle of ψ dilatancy equals the internal angle of friction Φ, contradicting test results. A correction, where the stress condition on Mohr's circle for stresses is dictated by ψ rather than Φ, was used for this calculation. This corresponds to a reduced angle of friction Φ_d determined by the expression:

$$\tan \Phi_d = (\sin \Phi \cos \psi) / (1 - \sin \Phi \sin \psi) \qquad (2)$$

▲ *Fig. 4.1-10 Excavation of crushed stone in trial pit*

The question of shear transfer through the crushed stone wedge to the clay till interface was further addressed by conceptual, small scale laboratory model tests. Analyses of the test results, in particular the displacement patterns, and comparisons with ABAQUS finite element simulations, confirmed the relevance of the expression for this problem.

Trial compaction

It was considered difficult to devise proper methods for in situ compaction control of the crushed stone beds under the construction elements. Instead, a method prescription system was adopted for placing and compacting the crushed stone. To verify this system, a trial pit enclosed by sheet piling was established onshore for underwater, full-scale compaction trials (see Figure 4.1-10).

Material excavated from the trial pit, after compaction and lowering of the water table, was used for the large-scale laboratory tests.

Clay till (emphasis on disturbed state)

The bearing capacity of failure mode 2 (see Figure 4.1-7) is given by the geometry of the rupture figure and the strength of the transition zone between the gravel and the intact clay till. The excavation in the clay till ahead of placing and compacting the crushed stone in the gravel wedges will inevitably disturb/remould the topmost part of the clay till.

Determining the strength of this zone was one of the key parameters in designing the anchor blocks. Before a final conclusion could be drawn, a comprehensive test program was decreed necessary, containing tests in the field as well as tests in the laboratory. These field tests comprised 28 sliding tests with 1m × 2m concrete blocks placed on clay till with different degrees of disturbance and loaded with different displacement rates.

The laboratory tests comprised more than 70 interface tests in the purpose-built large sliding box. The tests were carried out on 400mm cylindrical and 100mm high reconstituted clay till specimens. For comparison, 20 conventional direct shear box tests on 100 by 100mm^2 and 30mm high reconstituted clay till specimens were carried out too.

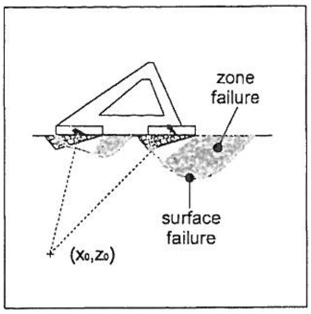
The effects from consolidation, ageing, pre-shearing, and displacement rates were tested.

Foundation reliability, a probabilistic approach

Although extensive research during the last two to three decades have matured the field of probabilistic analysis for practical applications, it is still considered to be an exceptional discipline within the present practice of structural engineering. However, in outstanding projects like the East Bridge traditional methods of structural engineering are often taken beyond the limits of previous experience. This calls for rational ways to extrapolate them safely, and in several situations probabilistic methods have proved indispensible.

One of the more ambitious applications of probabilistic analysis was to verify the reliability of the anchor block design, which was developed according to traditional geotechnical design methods. This particular case was interesting due to the large amount of geotechnical test results, which can be represented by a stochastic model.

In the assessment of the foundation reliability the soil capacity is based on the upper bound theorem of plasticity and hence on the work balance of an imposed failure motion. Two different failure modes are considered: the first assumes activation of the clay in zone failures emerging from the stone wedges, while the second mode assumes a sliding failure in the wedge and clay interface layer. These correspond to failure mode 3 and 2, respectively, shown in Figure 4.1-7.



▲ *Fig. 4.1-11 Zone failure geometry*

Zone failure

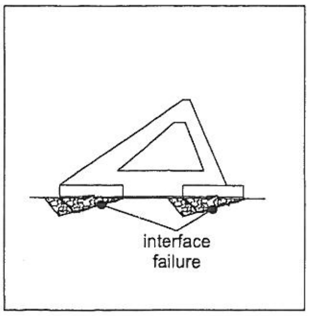
For the zone failure mode, the anchor block is assumed to move as a stiff body in a plane, rotational motion, specified by the co-ordinates of the centre of rotation. With assistance from professor Bent Hansen of the Technical University of Denmark, a consistent soil failure description was developed including surface failures in the stone wedges, and surface and zone failures in the clay till.

The work balance comprised the internal plastic dissipation in the stone wedges and the clay till, minus the external forces from the cable and the gravity of the anchor block. Due to the probabilistic representation of the soil strength, the work balance is a random variable and thus defines a classical reliability problem: to determine the probability of failure, ie that the work balance is negative. Determination of the distribution of the internal work involves integrals of the soil strength model over up to six independent dimensions, and reliability calculation of a failure mode for an assumed centre of rotation is thus far from trivial. The reliability is a function of the assumed centre of rotation, and an optimisation is required to find the most critical centre – i.e. with the lowest reliability. Due to the extensive calculations required for each reliability calculation, the optimisation was performed manually.

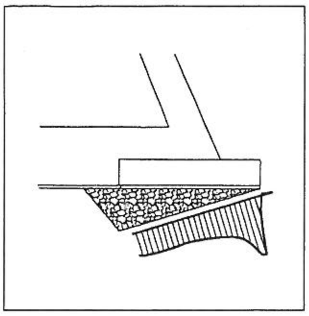
The primary result of the zone failure study was a minimum estimate of the reliability of $\beta = 4.9$. This corresponds to the high safety class in the Danish codes and is appropriate for the anchor block foundation. The influence on the reliability estimates when various simplifications of the soil model were employed was also analysed, to document the influence of the very detailed utilisation of the geotechnical test results. Based upon these variations it was concluded that modelling of the strength increase due to consolidation was of minor importance, while the detailed reproduction of the CPT records (conditional mean value) and the associated reduction in uncertainty (conditional variance) near observation points, was of major importance. Hence, with all three features eliminated, the reliability estimate dropped to $\beta \sim 4.0$. This corresponded to normal or low safety class, which is not considered sufficient for the anchor block foundations.

Sliding failure

The clay till surface of the excavation for the stone wedges will be remoulded and a weak interface between the wedge and the intact clay is thus expected. Hence, a



▲ *Fig. 4.1-12 Sliding failure in stone wedge interface layer*



▲ *Fig.4.1-13 Consolidation stresses in interface layer.*

sliding failure in this weak interface layer has to be considered in assessing the capacity and reliability of the foundation.

Some strength recovery is expected due to reconsolidation and pre-shear when the foundation is loaded, and also due to ageing. A dedicated geotechnical test programme has been carried out to quantify these effects. The compiled test results enable a direct probabilistic formulation of the consolidation and ageing effects, with due consideration of both statistical uncertainty and inherent stochastic variations.

The test results concluded the strength increase due to consolidation to be of primary importance, and it was thus essential to establish a detailed description of the stress distribution in the interface layer. This was accomplished with FEM model calculations, using a model involving the wedge, the interface layer, and the elastic halfspace of intact clay till. The FEM analysis indicated a peak in the stress components at the toe of each foundation pad.

From approximations of this stress distribution, the shear strength variation in the interface layer was determined, enabling the internal dissipation in a sliding failure to be calculated by numerical integration. Similar to the zone failure assessment, the reliability assessment was based on a limit state defined as the balance between the plastic dissipation in the interface zone and the external work.

The assessment documented a very high reliability $\beta = 10$, and a relatively moderate influence of consolidation and ageing was observed. The results indicate a high sensitivity to the inclination of the stone wedge, and a reduction from the tender design value of 15° to 10°, resulted in a more appropriate reliability level ($\beta = 5$). However, a reduction in the inclination would also influence the reliability of the zone failure mechanism, so the value of 15° was maintained.

Settlement of the anchor block

Using advanced calculation methods was the only possible means to calculate the direction and size of the settlement of the anchor block. By using the finite element program ABAQUS it was possible to include the effects of the special geometry of the anchor blocks and the gravel wedges as well as the limited thickness of the clay till, and the loading sequence of the anchor block.

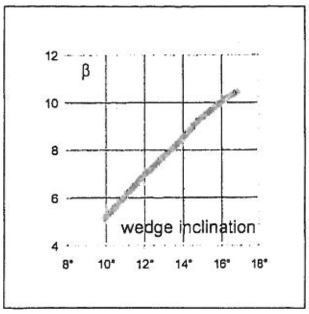
Oedometer and triaxial test results formed the basis for determining the soil deformation parameters.

Pylons

Each 254m high pylon has a base 78m long and 35m wide, dimensions which were primarily dictated by the requirements for structural integrity in the base and in the lower part of the pylon leg, and secondarily by bearing capacity and settlement criteria.

A special task involved assessing the risk of liquefaction in the soil below the base due to wind loading during construction of the pylons. Analyses based on traditional liquefaction models, as well as a recently published new liquefaction model, showed this risk to be negligible.

The very high friction angles for the crushed stone bed material gave rise to some concern in connection with placing the precast lower parts of the pylons. The question was whether sufficient skirt penetration would be achieved, but by placing a screeded, looser layer at the top of the stone bed, the predicted load-penetration response was achieved with full skirt penetration and desired base contact prior to grouting the concrete-stone interface.



▲ *Fig. 4.1-14 Reliability of sliding failure as function of wedge inclination*

Approach piers

The bases of the approach piers are typically 23m long and 19m wide. Bearing capacity analyses in the limit states ULS and ALS were treated in a top-down manner. First a simple analysis, with the weakest soil parameters from the design soil profile (see Figure 4.1-2), was carried out for all piers with their individual geometries and loads. Then for those particular piers with bearing capacity problems, a more refined analysis was applied taking the spatial variation of the soil strength into consideration.

All piers in all load cases and projections were analysed in one run according to DS 415. Load and geometry were shared in a database with the structural engineers. Whenever data changed, a new analysis could be performed swiftly without retyping a lot of data. The weakest soil strength within a depth corresponding to the width of the footing was used as a conservative first estimate representative for each pier. Those piers not fulfilling DS 415 (1984) with this approach were analysed with a method applying the local soil strength at each part of the rupture figure for the critical load cases.

Ship impact analyses were addressed within a framework of probabilistic analyses. The consequences of impacts from ships of different sizes and velocities were charted, with a computer program (SIAS62) to generate results for the probabilistic analyses. The ship impact analysis was performed taking the total construction into account: pier, bridge girder, neighbouring piers, and soil, thus permitting the whole construction to absorb an impact. The soil is modelled by simple linear elastic, ideally plastic springs for deflection in three directions as well as in torsion. These springs only model soil layers in relatively close contact with the foundation, whereas the far field effects are not modelled.

For the soil part of the analysis, an independent 3D finite element analysis using the FENRIS program was decided upon, since a control calculation by hand was considered prohibitive. The results of the SIAS62 and the FENRIS simulations were in very good agreement.

The challenge of the SLS analysis was to perform an analysis true to the nature of the clay till. A calculation method was prepared, fully implementing the SHANSEP strength and stress model with Bjerrum's concept of primary and secondary consolidation. The application of the different loading-unloading schemes during the different stages of construction could be modelled, while at all times keeping track of consolidated strength, preconsolidation, and displacements.

A database was developed with the design soil profiles for the piers. Also a reference to the loads and geometries database was used. Then, for each pier, a load-settlement prediction could be carried out. All layers, down to the marl, were modelled with settlement parameters representative of the soil type and its initial stress and strength data. For the clay till a dramatic increase in settlement (creep) occurred when a limit of approximately 70% of the preconsolidation stress, σ'_{pc}, was exceeded.

Approach ramps

In Halsskov on Zealand the natural topography allows almost direct access to bridge level, whereas a man-made embankment rising some 24m above sea level was necessary on Sprogø. This, however, introduced a unique possibility for full-scale testing of the settlement and creep model established for the clay till, based on laboratory tests from the West and East Bridge projects. Settlement gauges were installed in the base of the embankment, in the underlying clay till, and finally below the clay till in the marl. Monitoring of these gauges is still on-going.

Lessons learned

The experiences from this project may be summarised as follows:

- A comprehensive geological model is necessary for proper correlation of even the most detailed and advanced testing programme
- Modern data handling and statistical methods are required for such complicated conditions and these may lead to much more rational design procedures
- CPT testing is a good tool for obtaining detailed strength information with depth for the encountered till formations
- Large scale laboratory tests and in situ tests are absolutely necessary
- Compacted layers of crushed rock gravel only show dilatancy very late in a failure process at high stress level. Such a layer may in this context be considered ductile
- Control of skirt penetration into gravel pads of crushed rock is feasible by introduction of an upper, uncompacted gravel layer with a carefully selected thickness
- Remoulded clay till will gain an undrained shear strength of 0.35 times the consolidation pressure for normal loading rates (0.45 for high loading rates). This strength includes the effects of consolidation and ageing but not any effect of shear deformations.

In conclusion, the over-riding message from the project is that very close co-operation between geotechnical engineers, structural engineers and engineering geologists is a prerequisite for success. Only by interaction within the different disciplines and by testing in different scales, numerically as well as physically, can we be sure to obtain safe and cost effective solutions and hope to advance the state of the art.

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